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Testing and Evaluation of Morton Creek Bridge

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Abstract:
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Key Words: Morton Creek Bridge Test, bridge testing, reinforced concrete slab-on-girder bridge

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1/ INTRODUCTION

Morton Creek Bridge on King's Highway 15, District 8, Kingston, was constructed in the year 1934 by the then Department of Highways, Ontario. The superstructure is a three-span cantilever and suspended span structure consisting of concrete deck slab supported by a series of concrete-encased steel angle lattice girders. The concrete deck of this bridge was waterproofed and repaired in 1975 and a deck condition survey was carried out by the Ministry in 1980. The condition survey report indicated considerable spalling of the concrete encasement to the lattice girders. It was recommended that a structural evaluation and/or load testing of the bridge be carried out. A more recent inspection of the bridge has shown worsening condition of the roadway beam undersides and highly pronounced rusting of the bottom chord angles of the lattice girders. The observed deterioration of the bridge deck elements, and the obvious fact that these older bridges are now expected to carry considerably higher live loads than those for which they were originally designed, necessitated a structural evaluation and load tests of the bridge. This report describes the structural evaluation and load tests performed on Morton Creek Bridge in August 1983.

2/ BRIDGE DETAILS

Morton Creek Bridge (shown in Figure 1) is a three-span reinforced concrete slab-on-girder type structure. The bridge is 36.1 m long and 11.6 m wide with two 4.57 m wide roadway lanes and a 1.52 m wide sidewalk, as shown in Figure 2. The bridge has a central suspended span 8.2 m long, supported by cantilevers continuous with the end span girders. The roadway deck slab is 177 mm deep, while the sidewalk slab is 100 mm deep and the deck slab is made composite with steel angle latticed girders beams encased in concrete. The bottom chord of the lattice girder follows a parabolic profile with the crown located at each midspan location. The girder depth varies from a maximum of 1150 mm at supports to 625 mm at midspan for both interior and exterior spans. The shear key for the suspended span was constructed by welding a pair of 9.5 mm thick, bent plates to the angle lattice girders. Adequate bond of concrete encasement to the plates was achieved through drilled holes in the plates. The two noteworthy features of this bridge are:

- the use of steel angle lattice girders for concrete reinforcements as well as the formwork support to construct the superstructure, and
- the use of welded shear plates to provide shear key for the suspended/cantilever span arrangement (see Figure 3).

The substructure consists of mass concrete piers and abutments, supported on timber piles.

Over the years, the Ministry has endeavoured to maintain the structure. In 1975 the deck was repaired by removing the deteriorated original non air-entrained concrete and then topping with air-entrained concrete to bring the overall slab thickness to approximately 240 mm. The deck was also waterproofed and covered by a bituminous wearing course. A condition survey [1] of the bridge deck in 1980 indicated the following:

- The new air-entrained concrete portion of the deck is in good condition while the old, non air-entrained concrete is showing some degree of deterioration with active corrosion near both deck ends.

- Considerable spalling of the encased concrete to the steel lattice girders and prounounced corrosion of the lattice girder steel angles has occurred.

- No wet areas were found on the underside of the deck confirming that the ingress of surface water into the deck has been halted by the resurfacing of the deck. However, Figure 4 illustrates the degree of deterioration observed on the roadway girders and the shear key locations before the structural evaluation and proof load test of the bridge was undertaken.

3/ BRIDGE EVALUATION

The bridge superstructure was evaluated according to the Ontario Highway Bridge Design Code (OHBDC) [2]. The deck slab capacity calculated using the simplified method given in Section 14 OHBDC is illustrated in Table 1.

As shown in Table 1, the minimum slab resistance is found to be approximately 1700 kN as opposed to the minimum required resistance of 420 kN for unposted bridges in Ontario.

The unfactored force effects on the roadway beam critical sections, shown in Figure 5, due to dead, superimposed dead loads and the OHBDC live loads are given in Table 2. The dead load force effects on the roadway beams where obtained by using simple beam analogy since the lattice girder was used initially as a supporting frame for the wet concrete of the deck and the girder. The superimposed load force effects due to the new concrete topping and the bituminous wearing course were also obtained by simple beam analysis.

Although more accurate values for the superimposed dead load forces could have been obtained by using grillage analogy, because of the relative magnitiude of the superimposed dead load forces involved, the error incurred by the simple beam analysis is considered negligible. The live load force effects on the bridge deck due to the OHBDC truck were obtained using the grillage analysis technique using the "GRIDS" computer program [3]. The grillage used for the analysis of the force effects due to OHBDC truck and the test vehicles used in this study is shown in Figure 5.

The truck was positioned longitudinally along the bridge to give the maximum force effects on the critical beam sections. The transverse positions of the trucks used to obtain the maximum force effect are illustrated in Figure 6.

The member resistance values at girder critical sections were also obtained using OHBDC. The dead load forces on the lattice girder were

induced well before the composite action of steel/concrete could be realised. Usually, the dead load stresses on the lattice girder need to be calculated separately using non-composite lattice girder properties and added to the stresses obtained on the composite section, due to the superimposed dead load and live loads, to check section adequacy. However, here the member forces for all loads were obtained using girder composite section properties, factored and summed directly to give the total ultimate forces on the critical sections and then compared with the ultimate member capacities (see Table 3). A load factor of 1.2 was used for the superimposed dead load since the depth of asphalt was determined reasonably accurately by the condition survey. The superstructure has a first flexural frequency of approximately 7.5 Hz for the end spans and 11 Hz for the suspended span which give a maximum impact factor value for this bridge of 0.25 as per the OHBDC.

However, a minimum impact factor of 0.3 specified in the Code to obtain the ultimate loads on the critical section was finally used. As evident in Table 3, apart from the centre section of the suspended span, the girders have adequate capacity to carry the live loads prescribed in the OHBDC without posting and the suspended span centre section has only 6% lower strength value than that required for the OHBDC live loads. The sub-structure appeared in good condition and therefore no attempt was made to evaluate the substructure.

4/ INSTRUMENTATION AND TEST LOADING

The load test on the bridge deck normally involves subjecting the deck to near ultimate load levels hypothesized to be generated by the OHBDC live loads. It is therefore very important that the bridge be monitored closely and thoroughly during the actual test. The response of the structure was monitored by strategically located deflection transducers, uniaxial strain gauges (both weldable and demountable type) and 45° rosette strain gauge (see Figure 7). Twenty-two displacement transducers, 44 uniaxial strain gauges, and a 45° rosette were used -- mainly distributed around the critical sections of Girder Nos. 1, 2 and 3. The weldable strain gauges were used around bottom chord locations where the concrete has either spalled or appeared to have a weak bond to the steel angles. Elsewhere demountable gauges were used.

The load during the test was applied by using two MTC test vehicles described by Bakht and Csagoly [4].

In order to load the bridge, two trucks were used, either individually or together, for truck loads up to approximately 665 kN and a simple truck was used for truck weights above 665 kN up to 890 kN. The two trucks loaded above 665 kN were not used simultaneously to load the bridge since it was found, using the procedure described by Bakht and Jaeger [5], that the probability of two trucks each exceeding 700 kN weight and being present in the bridge simultaneously to cause the worst effects in the next 10 years period is approximately 0.002. The calculations are illustrated in Appendix A. Table 4 compares the ultimate force effects due to a single OHBDC truck to the maximum forces induced by the single test truck loaded to 890 kN. It can be seen that the single test truck weighing 890 kN causes force effects on the bridge which are comparable to factored forces due to single OHBDC truck.

The weight of the vehicle was gradually increased and for each load levels the trucks were placed either individually or together at predetermined locations to give the critical effects (see Figure 8).

The data from the strain gauges and deflection transducers were recorded through the computer based multi-channel data acquisition system des-

cribed by Bakht and Csagoly [4]. The bridge underside also was checked for any undue cracking by initially marking the cracks at selected locations and monitoring the crack growth after application of loads, particularly at higher load levels (see Figure 9). The strain gauges were installed a week prior to the testing of the bridge.

5/ TEST RESULTS

The strain measurements made on various strain gauges during the test were initially plotted aginst the truck load increments used in the tests. Typical graphs are illustrated in Figure 10. Generally, the strain gauges which had recorded maximum strains around 50 μs and above showed linear relationships to the test truck load increments. The gauges which recorded lower strain values did not exhibit a definite trend due to relatively higher percentage of error involved in such measurements. The strain values recorded on various gauges for the test truck loads of approximately 665 kN and 890 kN are illustrated in Tables 5 and 6, respectively.

The gauges located at the bottom chord of the lattice girders showed generally greater strain values than the gauges located elsewhere in the girder. The interior midspan bottom chord gauges 56 and 75 recorded the maximum strain values of 280 μs and 321 μs, respectively, for single truck loading of 890 kN. While the exterior midspan bottom chord gauges 38, 78 and 62 gave strain values of the order 85 μs, about a third of that recorded in the interior midspan. Although the live load moments induced were approximately equal (Refer to Tables 2 and 4) and the section properties are the same, the measured strain values indicate that the exterior midspan section is considerably stiffer than the interior midspan section.

At pier sections, contrary to general expectations, the bottom chord gauges 44 and 83 gave strain values (maximum 102 μs) much greater than the top chord gauges No. 63 and 41 (maximum 10 μs). It was thought that since the pier section has considerable reserve capacity to sustain the applied loads (Table 3), and the fact that the top steel reinforcement can help to sustain part of the applied moment, the strains recorded at the top were relatively lower. As evident from the Tables 5 and 6, the gauges used to measure tensile strains induced by the shear force showed relatively low strain values. At pier supports the maximum recorded strain was only 7 μs. Similarly, the gauges at suspended span supports showed very low strain values with a recorded maximum of 28 μs. These observed lower strain values consolidate the earlier observations made by

the deck evaluation procedure (Table 3) that these sections have considerable reserve capacity to carry the OHBDC live load shear forces.

Table 7 illustrates the maximum bottom chord stresses measured during the load test. The largest tensile stress included was 64 MPa recorded at the interior midspan section of Girder 3. The maximum compressive stress induced at the pier support section of Girder 1 was 20 MPa. It is generally believed that for concrete slab-on-girder type bridges, the live load forces require approximately 40% of the section capacity and the balance is to cater for the dead/superimposed dead load forces. If this is the case, the steel capacity available to carry live loads for a steel type with F_y = 225 MPa is approximately 90 MPa. However, for the beam section at Position 5, the actual steel capacity available for live loads is approximately 70% full capacity (i.e., 157 MPa). Since the recorded maximum stress was 64 MPa, the bridge deck has the adequate capacity to carry test truck loads and hence the OHBDC live loads without any posting. The observed corrosion of the bottom chord sections has not hampered the useful functioning of the bridge deck -- the mere fact that the bridge was able to carry the test truck and exhibit linear load/ strain behaviour shows that the bridge has considerable reserve capacity to carry OHBDC live loads without posting.

Due to the observed cracking around the shear key of the suspended span (Figure 4) there was some concern raised on the capability of the shear key to sustain the live load forces. Also, it was feared that the shear key failure can be catastrophic as recently was the case for the bridge on Interstate 95, the Connecticut Thruway, in the USA. Initial analysis of the shear key indicated that it has substantially higher shear capacity (640 kN) than the beam element (351.3 kN) that it helps to carry. The crack patterns observed around the shear key areas during the test did not indicate any crack growth which would cast doubt on the capability of the shear key elements. The 45° rosette strain gauge attached to the shear plate of the Girder 1 recorded maximum individual strains of only +10 $\mu\varepsilon$ and -38 $\mu\varepsilon$ for the tests performed in this study. The maximum principal strains obtained were +30 $\mu\varepsilon$ and -40 $\mu\varepsilon$ with corresponding stress values of 6 MPa tensile and 8 MPa compressive which prove the fact that the shear plate has considerable reserve strength to carry the OHBDC live loads.

The girder deflections were observed for each loading case at each of the interior and exterior midspan beam sections. The test results showed maximum deflections at exterior midspan when the truck was positioned longitudinally at Position 2. The maximum deflections at interior midspan sections was observed when the truck was moved to Position 4. The maximum deflections measured at interior and exterior midspan sections for the two-truck loading case with trucks loaded to approximately 665 kN are illustrated in Figures 11 and 12. Figures 13 and 14 illustrate the maximum deflection observed at interior and exterior midspan sections for single truck loading of approximately 890 kN.

The maximum observed deflection for the two-truck case was 4.85 mm while the maximum observed for single truck loading was 4.40 mm. In both cases, maximum deflection was observed at the interior midspan. For the interior span, which gave the observed maximum deflection, the first flexural frequency value is approximately 11 Hz. The allowable static deflections as per the OHBDC for bridges with sidewalks and little pedestrian use is approximately 4 mm (Figure 2.6.2.2.2 of the OHBDC). Although the bridge was loaded to near ultimate load levels, the measured static deflection is of the same order as that calculated for lower serviceability Type I load levels (80% OHBDC truck plus dynamic allowance). Obviously, the bridge has the ability to function satisfactorily under service loads and the structure deflections are well within the pedestrian comfort criteria specified in the OHBDC.

The observed deflections are compared with deflections calculated using grillage analysis and corrected for any discrepancies in the initially-assumed elastic modulus for the grid elements. The actual elastic modulus of the deck was found to be 43.0 kN/mm^2 and the uncracked section properties were used for the grid analysis. As evident from Figures 10 to 14, the observed deflections compare very well with the calculated deflections. This again confirms the fact that the bridge exhibits linear elastic behaviour for the applied loads, the magnitude of which range well into OHBDC ultimate load levels.

6/ CONCLUSIONS

The bridge evaluation study indicates the following:

- The deck slab has adequate capacity to sustain OHBDC live loads without posting,
- Apart from the interior midspan sections, the beams have adequate capacity to carry the live loads without posting.
- The interior midspan sections has only marginally lower strength (6% lower) than required by the OHBDC.

The static load test performed on this bridge showed generally a linear strain-load characteristics. A few strain gauges, where the measured strains were small, did not exhibit a definite trend. The bottom chord gauge at interior midspan sections recorded much greater strain values than exterior midspan sections. Obviously, the exterior midspan section is considerably stiffer than the interior midspan section. In contrast to general expectations, bottom chord gauges at pier sections showed greater strain values than the corresponding top chord gauges, possibly because:

- the top steel reinforcement shares the induced strains, and
- the top of the girder is very close to the neutral axis due to the composite action.

The girder sections have considerable reserve capacity to sustain the applied live load and, hence, the OHBDC live loads. Gauges attached to monitor shear strains gave very low strain values indicating that the sections are not very critical for shear.

The maximum measured stress for the total test performed on the bridge was 64 MPa, the allowable live load stresses is calculated to be approximately 157 MPa, which proves that the deck structure has adequate capacity to carry the OHBDC live loads without posting. It appears that the degree of corrosion observed at the underside of the lattice girders and the fact that the interior midspan sections are slightly under- designed, has not altered the useful functioning of this bridge structure. The shear key behaviour monitored through a 45° rosette strain gauge, crack growth around the shear key shows that the shear key has adequate strength to sustain OHBDC live load forces which confirms the conclusion made by the initial evaluation study.

The girder deflections observed at the interior and exterior midspan beam sections indicate that the measured deflections are well within that required to satisfy OHBDC requirements. The measured deflections compare very well with the calculated deflections using grillage analysis. This also confirms earlier observation that the deck is well within its linear elastic range under applied test loads and the observed deterioration does not affect the useful functioning of the bridge.

Based on the above findings it is concluded that the bridge need not be replaced or strengthened due to structural adequacy consideration. Immediate maintenance, however, is required to prevent further corrosion of the steel angle lattice and the steel seats at the shear key locations.

REFERENCES

[1] Tiede, H.F., "Detailed Bridge Deck Survey of Morton Creek Bridge," Bridge Site #16-45, Highway 15, District 8, Eastern Region, Ministry of Transportation and Communications, Downsview, Ontario, 1980.

[2] Ontario Ministry of Transportation and Communications, Ontario Highway Bridge Design Code, Downsview, Ontario, 1979.

[3] Sen, R., "Program for the Grillage Analysis of the Slab on Pseudo-Slab Bridge Decks," Department of Environment, London, England, 1975.

[4] Bakht, B., Csagoly, P.F., "Bridge Testing," SSR-10, Ministry of Transportation and Communications, Downsview, Ontario, 1979.

[5] Bakht, B., Jaeger, L.G., "A Rational Procedure for Overweight Permits," to be published.

[6] Csagoly, P.F., Knobel, Z., "The 1979 Survey of Commercial Vehicle Weights in Ontario," RR-230, Ministry of Transportation and Communications, Downsview, Ontario, 1981.

Appendix A

Probability of truck weight exceeding 700 kN in Ontario, Using Csagoly and Knobel (6) $= 2.3 \times 10^{-4}$

Assumed truck traffic per day $= 1000$

Number of truck passing the bridge per day $1000 \times 2.3 \times 10^{-4}$ $= 0.23$

Assumed time interval for which the truck causes the worst effects $= 1$ s

Probability of a truck exceeding 700 kN being on the bridge in a particular 1 s interval in a day $\frac{0.23}{24 \times 60 \times 60}$ $= 2.66 \times 10^{-6}$

Probability of two trucks each exceeding 700 kN being on the bridge in the same 1 s interval in a day $(2.66 \times 10^{-6})^2$ $= 7.1 \times 10^{-12}$

Number of 1 s intervals in the next 10 year period $10 \times 365 \times 24 \times 60 \times 60$ $= 3.15 \times 10^{8}$

Total number of times two trucks each exceeding 700 kN being on the bridge in the same 1 s interval in the next 10 years $7.1 \times 10^{-12} \times 3.15 \times 10^{8}$ $= 0.002$

Table 1/ Slab Capacity Related to OHBDC (Cl.14.6.5.2.)

Slab Thickness, mm	Percentage Steel, %	Slab Resistance (Rn), kN	Min. Slab Resistance Required, kN
240	0.22	1700	420

Table 2/ Unfactored Force Effects on Roadway Beam Critical Sections Due to Dead, Superimposed Dead and OHBDC Live Loads

CRITICAL SECTION	DEAD LOAD EFFECTS Moment	Shear	SUPERIMPOSED DEAD LOAD EFFECTS Moment, kNm	Shear	LIVE LOAD EFFECTS* Moment, kNm	Shear
1	--	39.8	--	14.5	--	64.4
2	37.8	--	23.6	--	249.3	--
3	179.4	97.1	79.0	33.8	301.7	102.1
4	--	54.8	--	20.9	--	79.3
5	117.6	--	46.3	--	254.3	--

* Live Load Effects Factored by multiple truck presence reduction factor where appropriate.

Table 3/ Comparison of Factored Force Effects and Factored Resistances

CRITICAL SECTION	FACTORED FORCE EFFECTS Moment, kNm	Shear, kN	FACTORED RESISTANCES Moment, kNm	Shear, kN
1	--	182.4	--	460.0
2	527.4	--	620.0	--
3	859.2	342.9	1169.0	460.0
4	--	235.1	--	351.3
5	659.5	--	620.0	--

Table 4/ Comparison of Factored Forces due to Single OHBDC Truck To Maximum Forces Induced by Single Test Truck Weighing 890 kN

CRITICAL SECTION	UNFACTORED FORCES DUE TO SINGLE OHBDC TRUCK		FACTORED FORCES DUE TO SINGLE OHBDC TRUCK		MAXIMUM FORCES INDUCED BY SINGLE TEST TRUCK WEIGHING 890 kN	
	Moment, kNm	Shear, kN	Moment, kNm	Shear, kN	Moment, kNm	Shear, kN
1	--	57.5	--	104.7 (100.6)	--	98.3
2	156.5	--	284.8 (273.8)	--	279.2	--
3	241.4	79.6	439.3 (422.5)	144.8 (139.3)	442.5	156.1
4	--	55.6	--	101.2 (97.3)	--	89.9
5	171.1	--	311.5 (299.4)	--	293.8	--

( ) Using Impact Factor of 0.25

Table 5/ Strains Recorded in Various Gauges for the Truck Loads of Approximately 665 kN Each (Using 48 Blocks)

GAUGE NO.	GAUGE LOCATION	POSITION LINE	STRAIN x 10^6																									
			1					2					3					4					5					
			1	2	3	4	5	1	2	3	4	5	1	2	3	4	5	1	2	3	4	5	1	2	3	4	5
32	G1-1		1	0	0	0	-2	0	0	-1	-2	-2	1	0	-2	2	1	-1	1	-2	0	0	0	0	-2	1	0
38	G1-2		32	6	38	19	24	54	5	61	10	22	-22	-6	-31	-20	-26	-30	-9	-39	-22	-35	-17	-6	-25	-15	-22
40	G1-2		4	1	5	2	3	6	0	6	2	2	2	0	1	1	-3	0	0	1	1	1	-1	-1	2	-1	0
44	G1-3		-13	4	-11	7	5	-50	2	-49	8	-10	-56	-9	-65	-33	-42	-70	-9	-84	41	-55	-46	-7	-52	-26	-34
41	G1-3		-2	0	-3	-1	-1	-2	0	-3	0	-1	0	0	0	2	1	0	0	-1	2	2	-1	0	0	3	2
42	G1-3		-1	2	-1	-2	0	-1	1	0	-1	0	0	-1	1	-1	0	1	0	-1	-3	-2	0	0	-1	-2	-1
46	G1-4		-3	-3	-4	-10	0	3	0	-3	-3	-2	11	-1	7	23	14	11	2	19	28	16	4	0	16	25	17
56	G1-5		-11	-2	-15	-10	-12	15	0	15	7	6	170	14	195	73	92	133	16	156	73	93	175	13	198	71	92
78	G2-2		35	10	43	24	36	62	9	71	45	56	-12	-6	-20	-10	-18	-18	-8	-26	-14	-23	-12	-7	-19	-10	-16
58	G3-1		4	-1	2	3	1	4	-2	2	-3	1	-1	0	0	1	1	-1	0	0	0	-1	0	0	-1	1	-1
62	G3-2		23	24	47	29	49	34	28	62	56	84	-10	-7	-17	-8	-16	-12	-9	-22	11	-21	-8	-6	-14	9	-15
63	G3-3		-1	0	-2	-6	-6	2	1	4	4	5	8	1	10	10	9	9	1	10	10	9	8	1	11	7	7
83	G3-3		-17	-20	-30	-29	-40	-25	-23	-50	-44	-69	-29	-33	-62	-48	-76	-42	-42	-83	67	-107	-26	-29	-55	-41	-68
64	G3-3		2	1	3	-2	1	2	1	3	2	3	2	-1	4	2	0	3	0	1	0	0	1	0	1	0	1
68	G3-4		-1	3	1	0	1	1	2	1	-2	-1	4	0	0	0	-4	6	-3	0	3	-1	6	-5	-1	1	-3
75	G3-5		-9	-1	11	-3	-14	-1	-3	12	15	25	111	90	225	206	307	90	89	197	112	211	123	114	260	220	235

Gx-y

Gx = girder number

y = truck position

Note: Gauge numbers are given in Figure 7.

Table 6/ Strains Recorded in Various Gauges for the Truck Loads of Approximately 890 kN (Using 72 Blocks, Kenwork Truck)

GAUGE NO.	GAUGE LOCATION	POSITION LINE	STRAIN x 10^6 1			2			3			4			5		
			1	2	4	1	2	4	1	2	4	1	2	4	1	2	4
32	G1-1		2	0	-1	0	0	-2	0	0	0	0	0	-1	-1	0	-1
38	G1-2		48	11	32	80	10	33	-34	-5	-28	-43	-10	-33	-30	-6	-22
40	G1-2		5	0	2	8	1	2	4	0	0	3	1	0	2	0	0
44	G1-3		-19	8	5	-72	5	-14	-84	-8	-51	-102	-13	-66	-70	-7	-40
41	G1-3		0	0	0	-2	0	-1	1	0	1	1	0	1	-1	0	1
42	G1-3		-2	2	-1	-1	0	0	4	0	0	4	0	-1	2	0	-1
46	G1-4		-9	-3	7	-7	3	6	12	-4	19	24	-2	24	1	0	22
56	G1-5		-20	-5	-13	18	-2	6	245	18	11	199	22	109	280	18	113
78	G2-2		46	12	38	84	12	70	-17	-7	-17	-22	-12	-20	-16	-6	-13
58	G3-1		6	-3	3	6	-5	0	2	-2	0	0	-1	1	1	0	-1
62	G3-2		34	28	40	47	37	82	-12	-10	-14	-16	-14	-18	-10	-10	-11
63	G3-3		-15	-1	-8	0	1	4	9	1	10	9	1	10	8	1	9
83	G3-3		-22	-18	-36	-36	-31	-56	-39	-42	-63	-57	-55	-93	-32	-37	-55
64	G3-3		-2	4	0	1	3	7	4	-1	4	6	-2	1	3	0	1
68	G3-4		0	4	2	1	4	0	6	-4	-1	8	-4	2	7	-1	3
75	G3-5		-4	-5	-8	7	1	15	155	141	269	127	125	151	183	170	321

Gx-y

Gx = girder number

y = truck position

Note: Gauge numbers are given in Figure 7

Table 7/ Maximum Chord Stresses Induced During the Test

GAUGE NO.	GAUGE LOCATION	LOADING CASE	MEASURED STRAIN x 10^6	CHORD STRESS, MPa
38	G1-2	5-1-2	80	16
44	G1-3	5-1-4	-102	-20
56	G1-5	5-1-5	280	56
78	G2-2	5-1-2	84	17
62	G3-2	5-4-2	82	16
83	G3-3	5-4-4	-93	-19
75	G3-5	5-4-5	321	64

5 - 1 - 2
LOAD LINE POSITION

G1 - 2
GIRDER No. POSITION

Note: Gauge numbers are given in Figure 7.

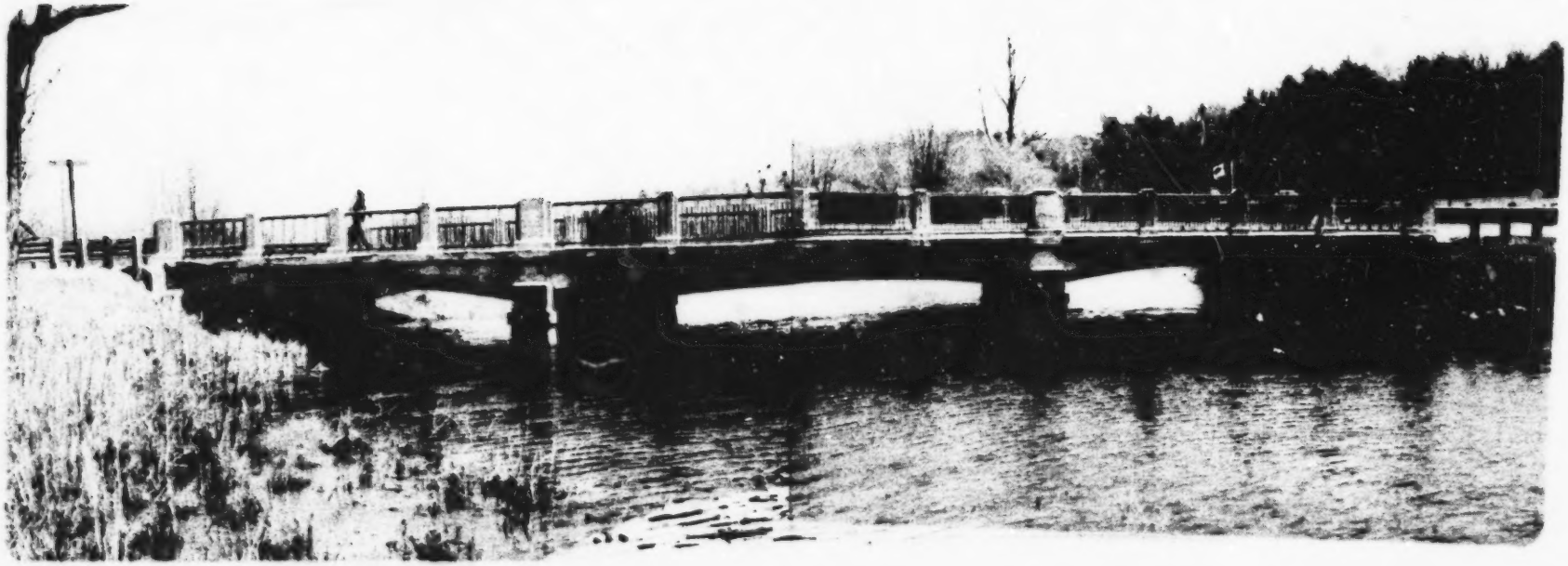


Figure 1/ View of Morton Creek Bridge

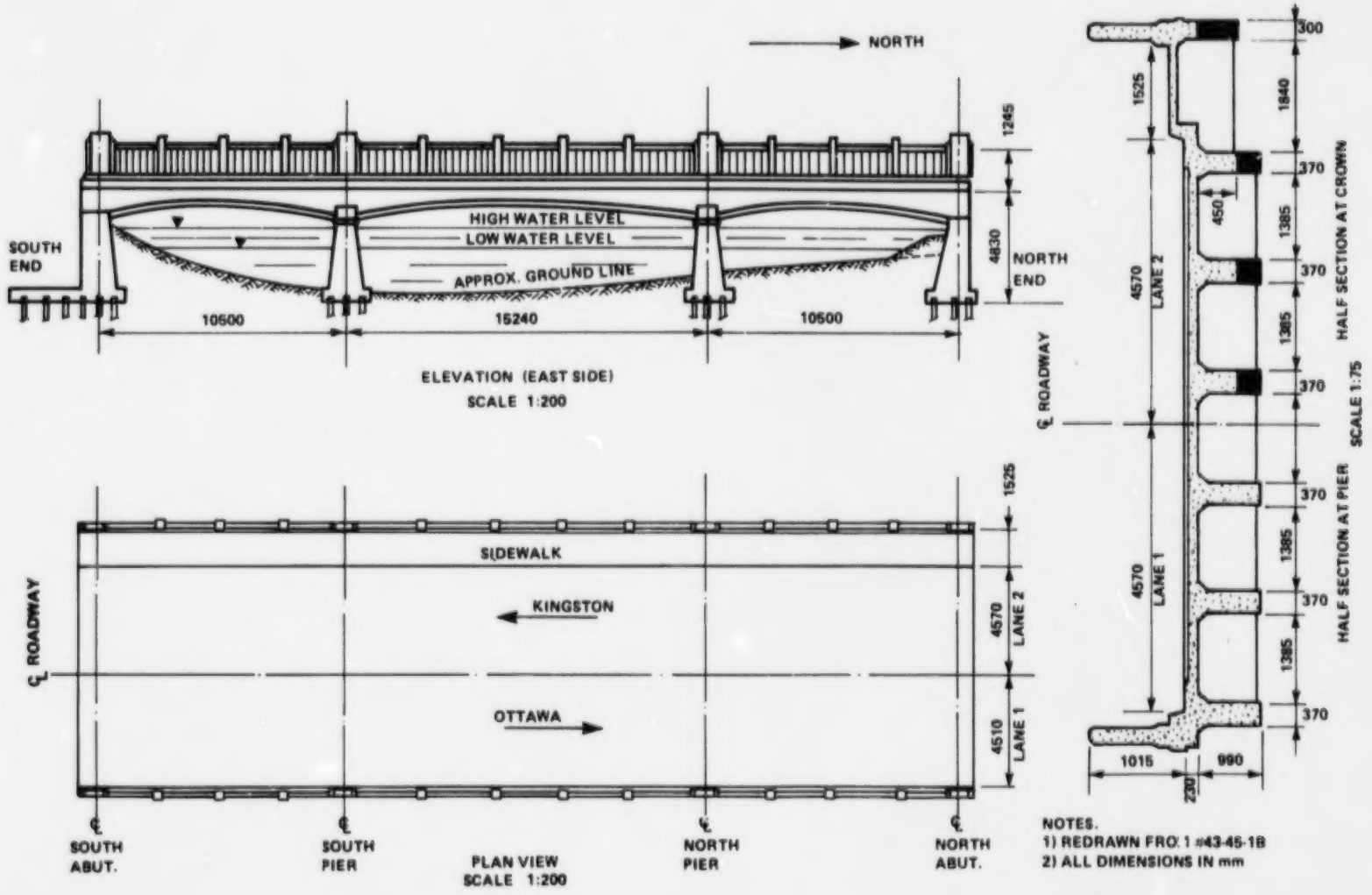
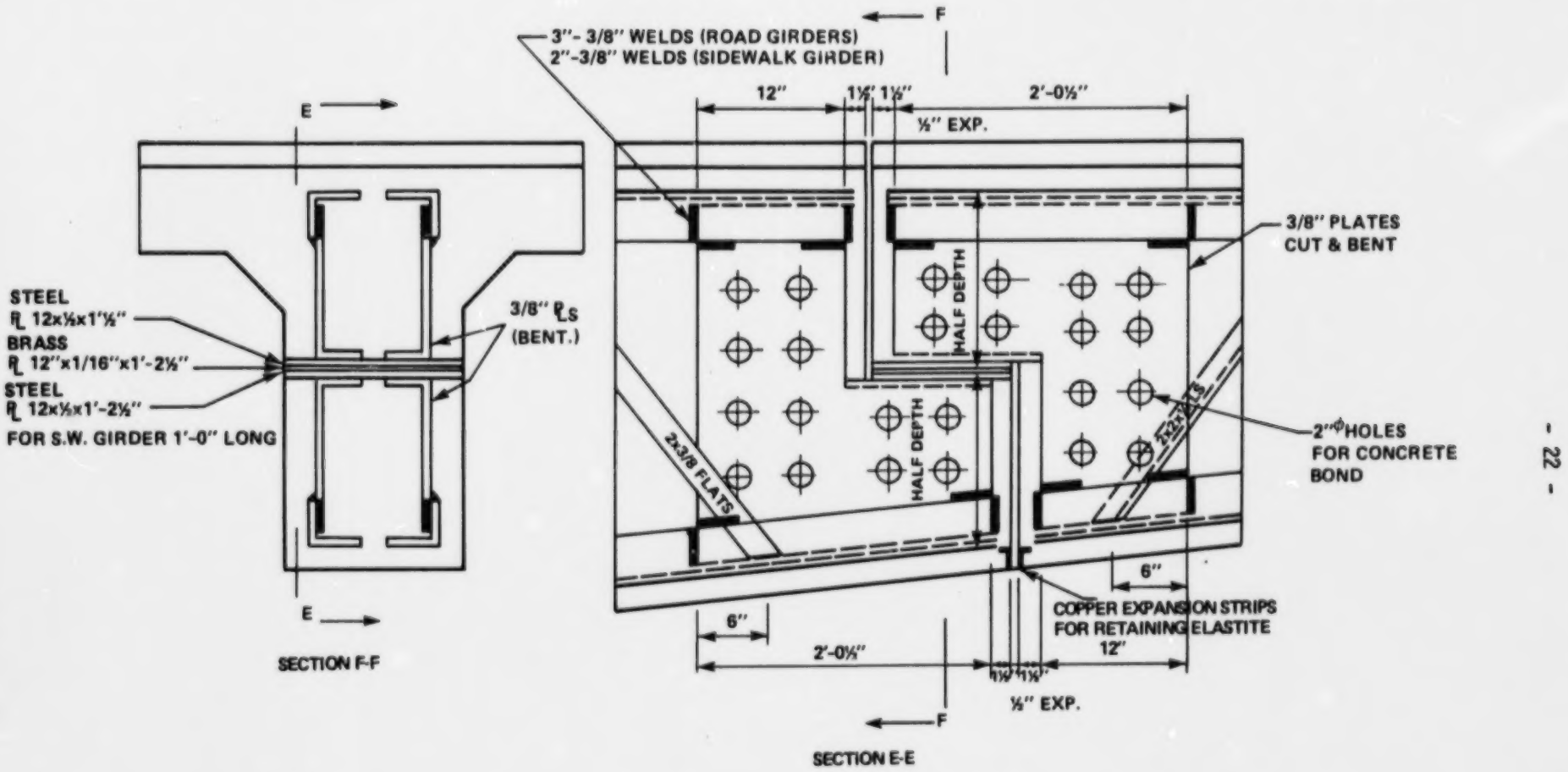


Figure 2/ Details of the Bridge



SCALE: 1 INCH = 1 FOOT

NOTE: Same principle for connections of sidewalk girder.
with difference in overall dimensions of plate
(see latticed girder)

Figure 3/ Detail of the Shear Key

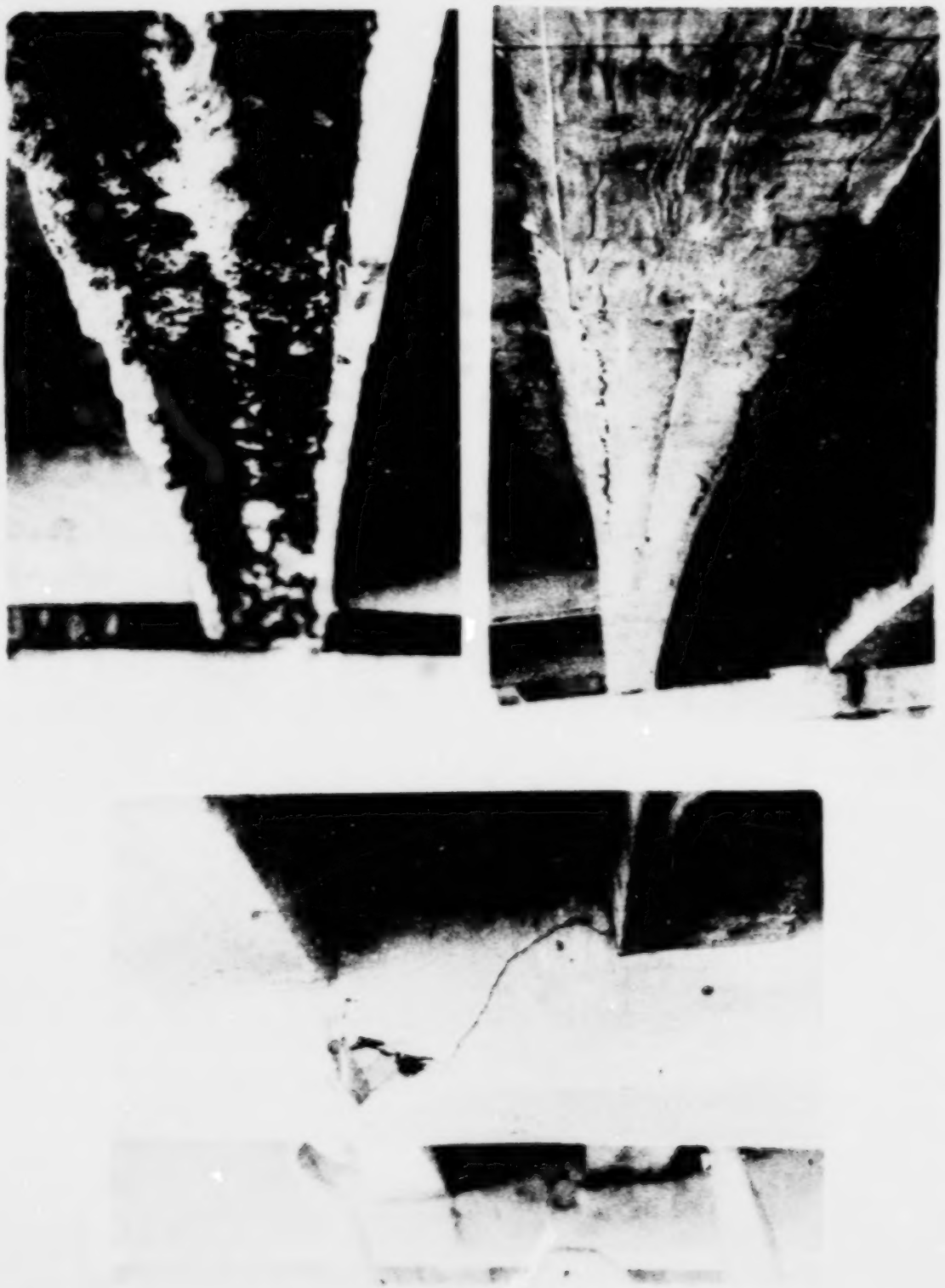


Figure 4/ Recently Observed Deck Deterioration

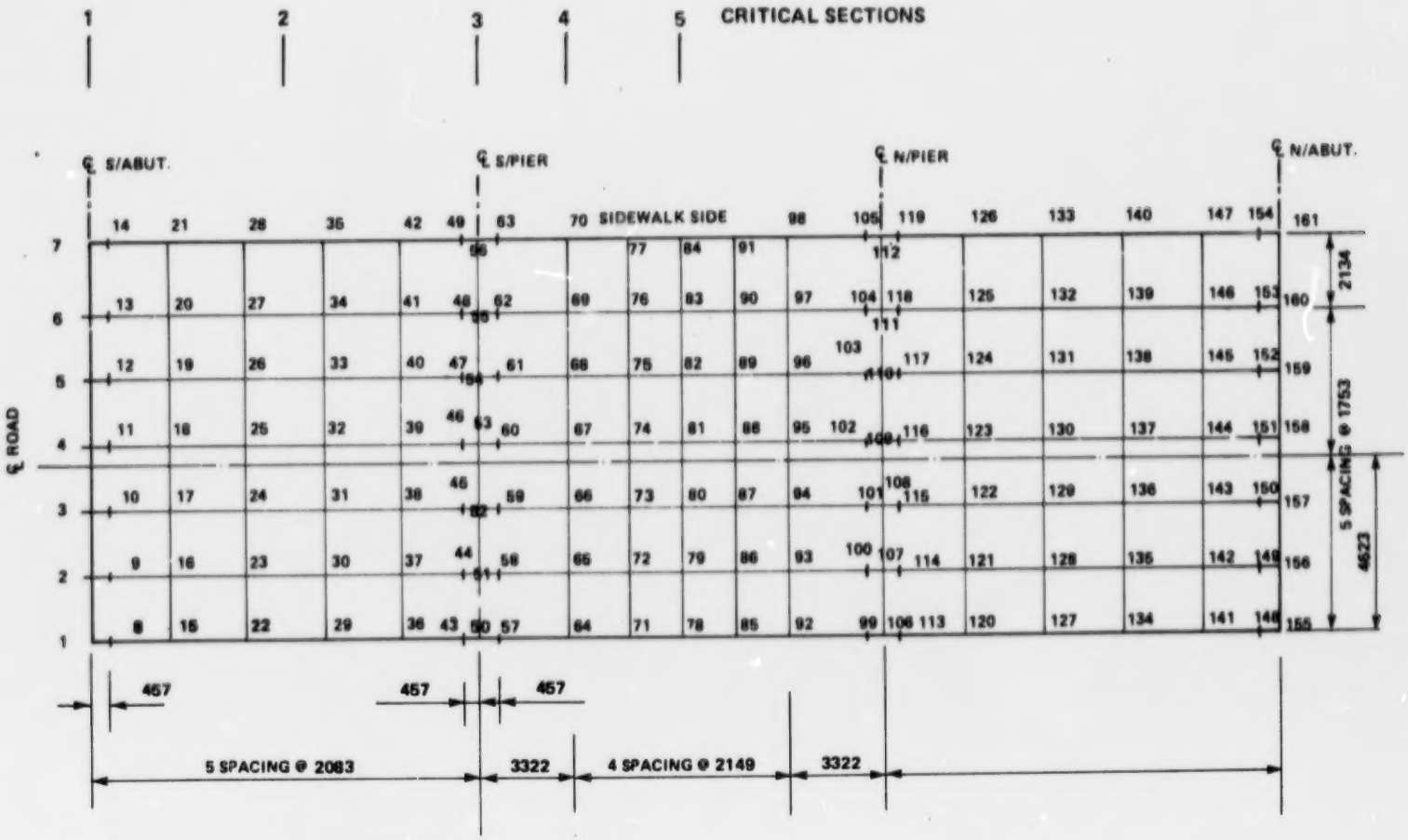


Figure 5/ Grids used for Deck Analysis

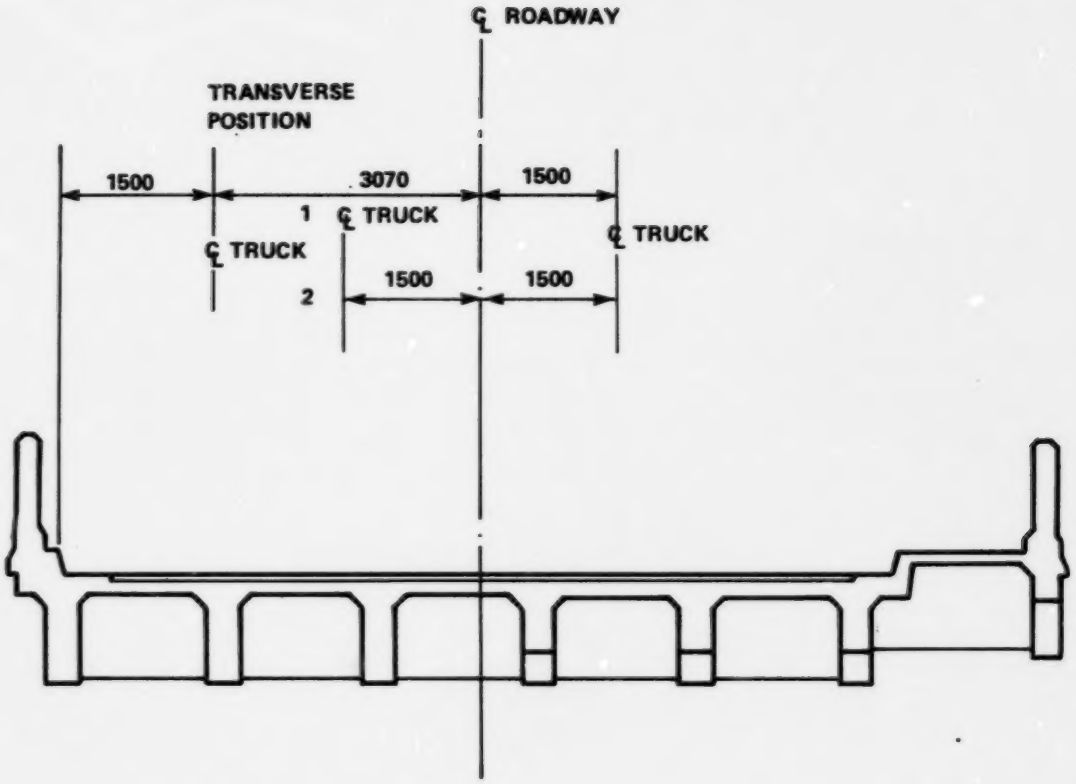
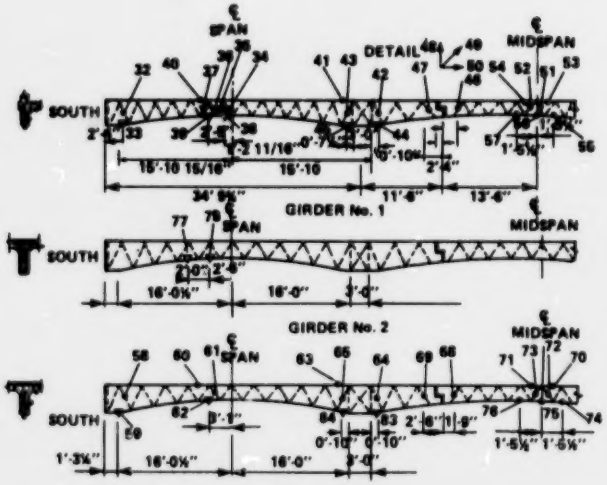


Figure 6/ Transverse Position of OHBDC Truck



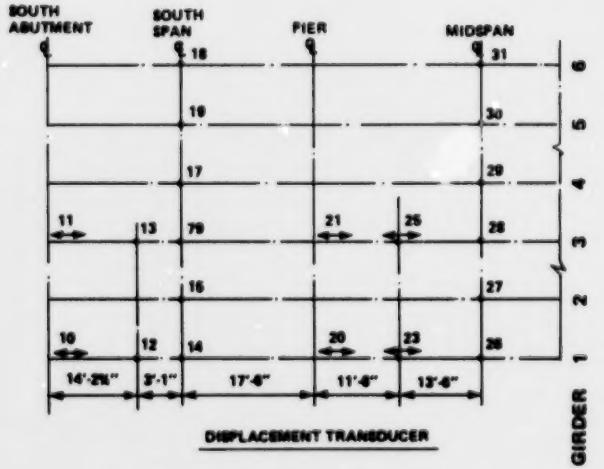


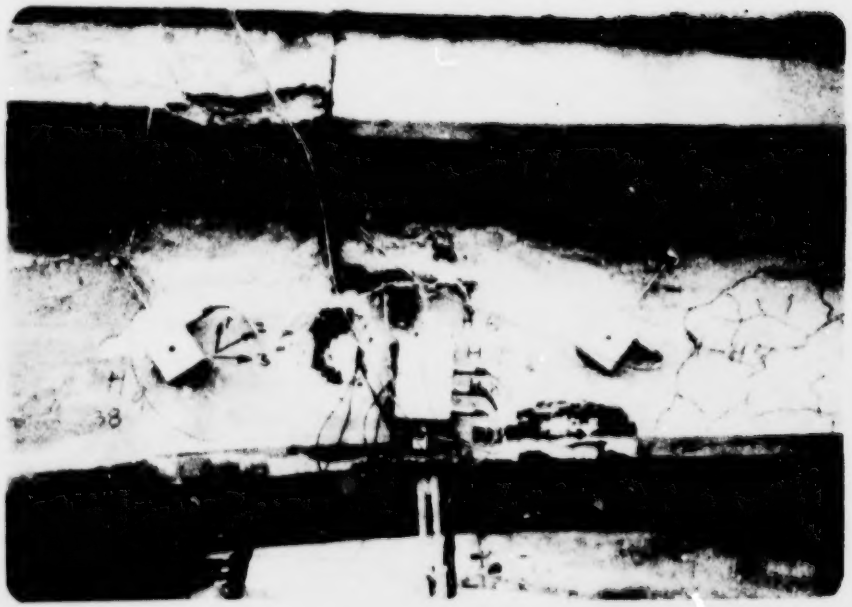
Figure 7/ Instrumentation Details

• DEMOUNTABLE STRAIN GAUGE (Installed on concrete)

▽ STRAIN GAUGE (Installed on concrete)

45° ROSETTE STRAIN GAUGE (Installed on plate)

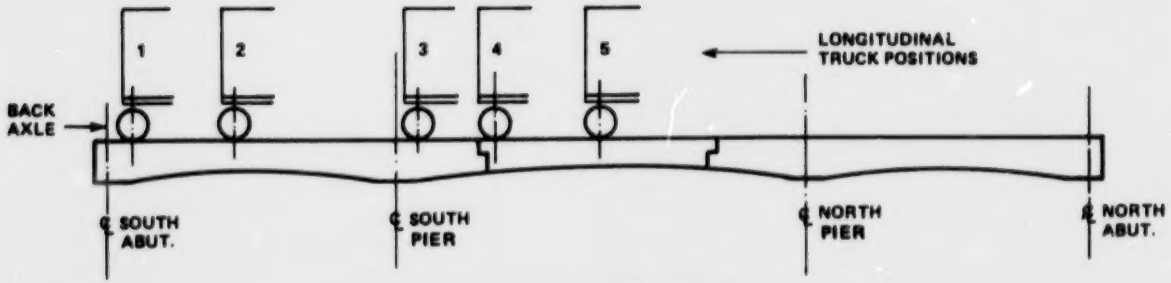
NOTE: THE SEQUENCE NUMBER 70 TO 79 WAS CHANGED DURING THE TEST TO 90 - 99.



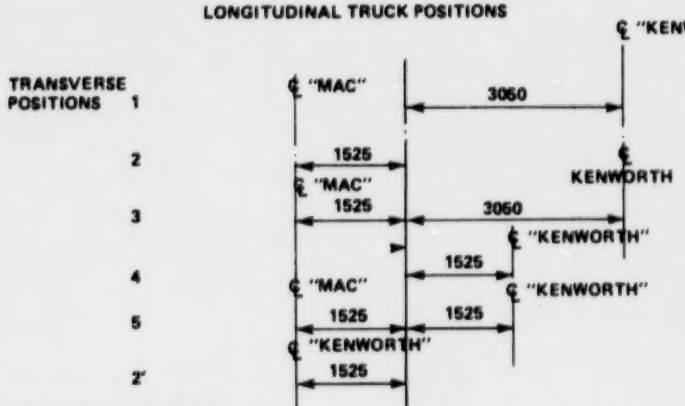
• VERTICAL DISPLACEMENT TRANSDUCER

↔ HORIZONTAL DISPLACEMENT TRANSDUCER

TYPICAL INSTRUMENTATION



LONGITUDINAL TRUCK POSITIONS

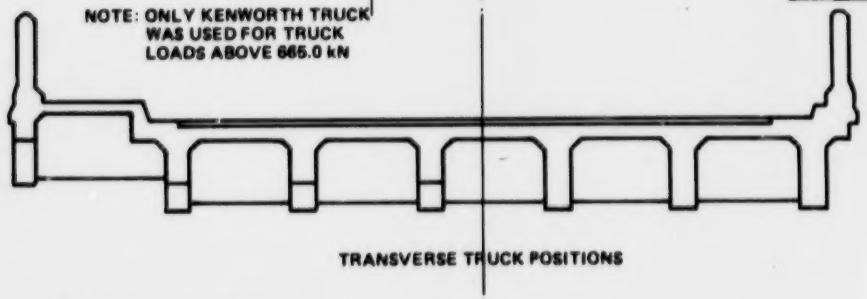


NOTE: ONLY KENWORTH TRUCK WAS USED FOR TRUCK LOADS ABOVE 665.0 kN

AXLE WEIGHTS OF THE TESTING VEHICLE

	LOAD LIFT No.	WEIGHT ON AXLE NO. (KN) 1	2	3	4	5	TOTAL VEHICLE WEIGHT (KN)
"KENWORTH"	EMPTY	43.6	46.4	46.4	41.0	41.0	218.4
	24 BLKS.	47.7	92.7	92.7	105.2	105.2	444.1
	36 BLKS.	49.5	115.5	115.9	136.9	136.9	554.3
	48 BLKS.	51.7	138.6	138.6	169.0	169.0	666.9
	60 BLKS.	53.5	161.9	161.9	201.1	201.1	779.5
	72 BLKS.	56.2	186.4	186.4	231.4	231.4	891.8
"MAC"	EMPTY	41.5	44.1	44.1	39.2	39.2	208.1
	24 BLKS.	43.7	90.0	90.0	104.7	104.7	433.1
	36 BLKS.	44.6	113.3	113.3	137.7	137.7	546.6
	48 BLKS.	46.9	136.0	136.0	170.3	170.3	658.5

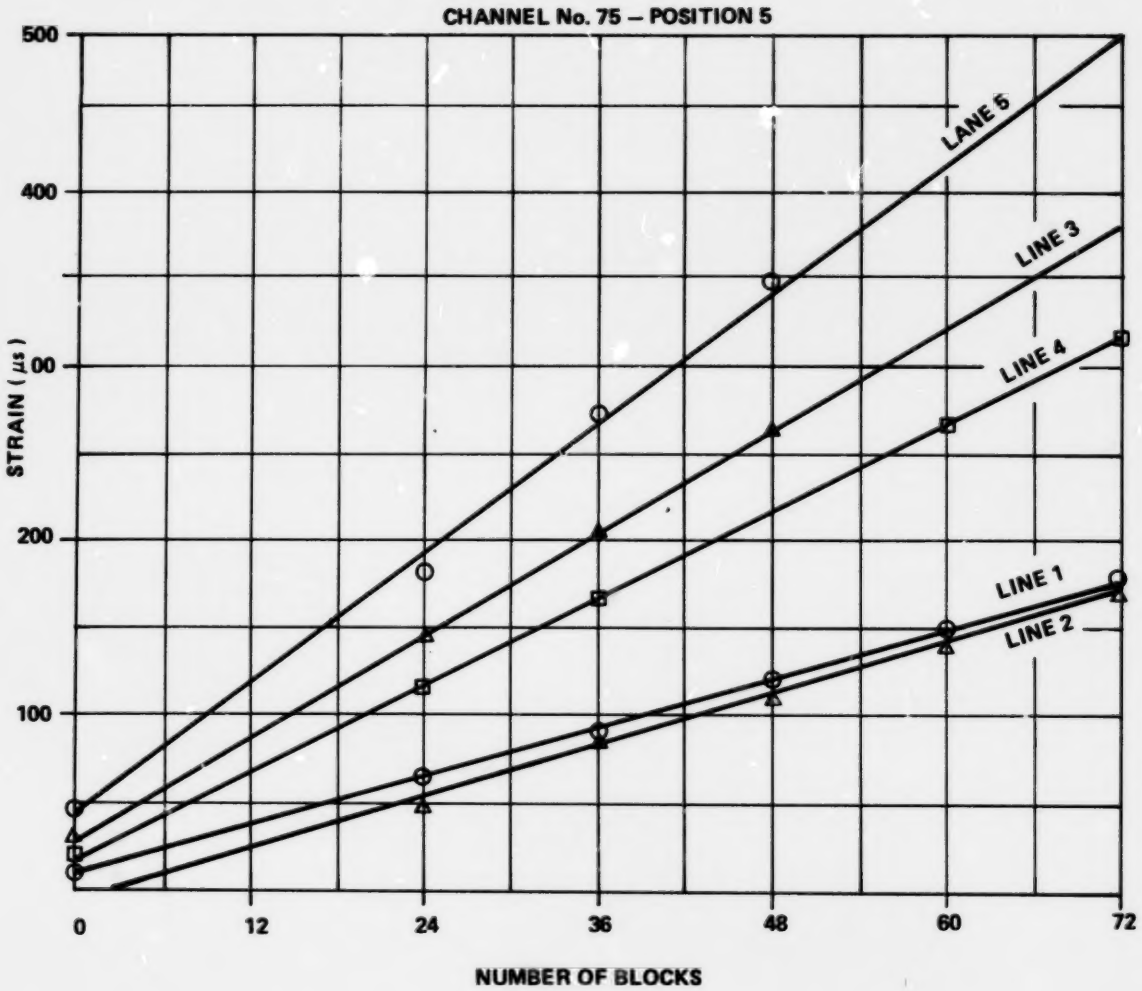
NOTE: LONGITUDINAL TRUCK POSITION ARE CALLED "POSITIONS" e.g. POS. 1, POS. 2, ...POS. 5. TRANSVERSE TRUCK POSITIONS ARE CALLED "LINE" e.g. LINE 1,.....LINE 5.



TRANSVERSE TRUCK POSITIONS

Figure 8/ Loading Truck Test Locations

Figure 9/ Monitoring of Crack Growth at Critical Sections



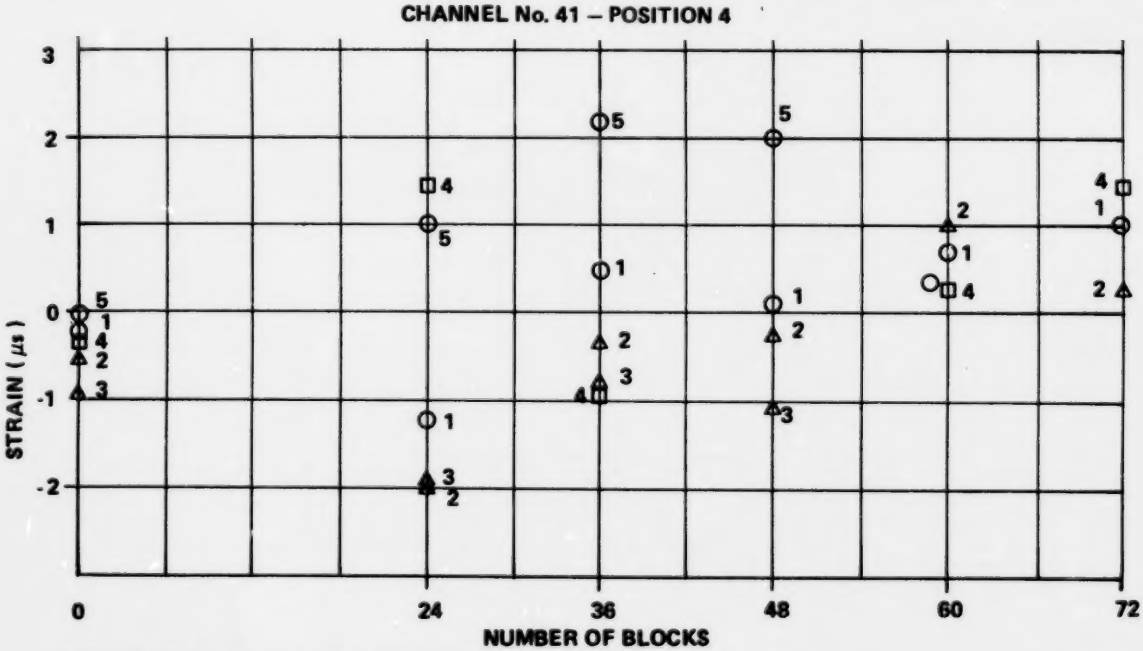


Figure 10/ Typical Strain Versus Load Increment Plot

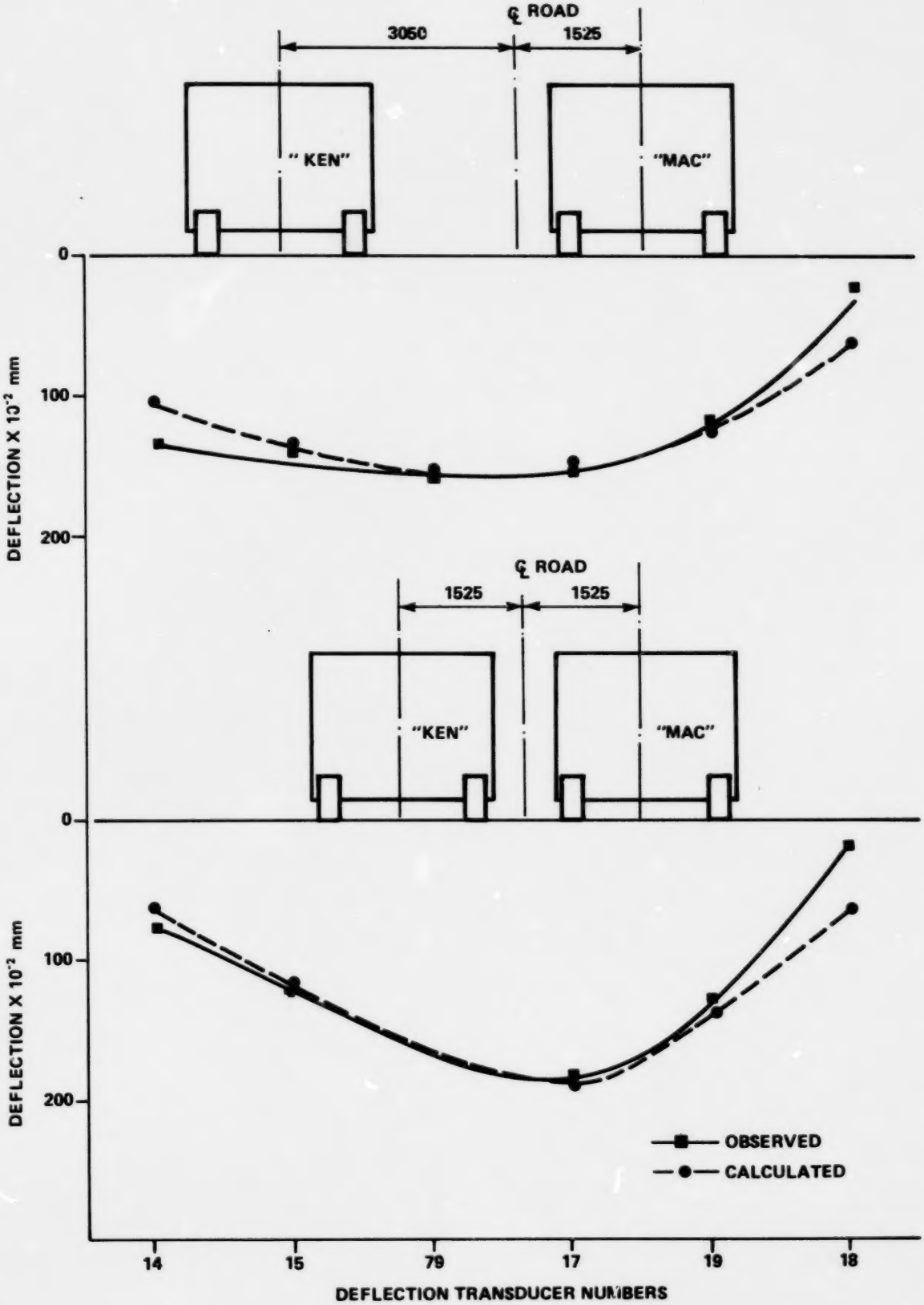


Figure 11/ Comparison ofMaximum Observed Deflections and Calculated Deflection at Exterior Midspan Due to the Trucks Each Weighing 665 kN (48 Blocks Each)

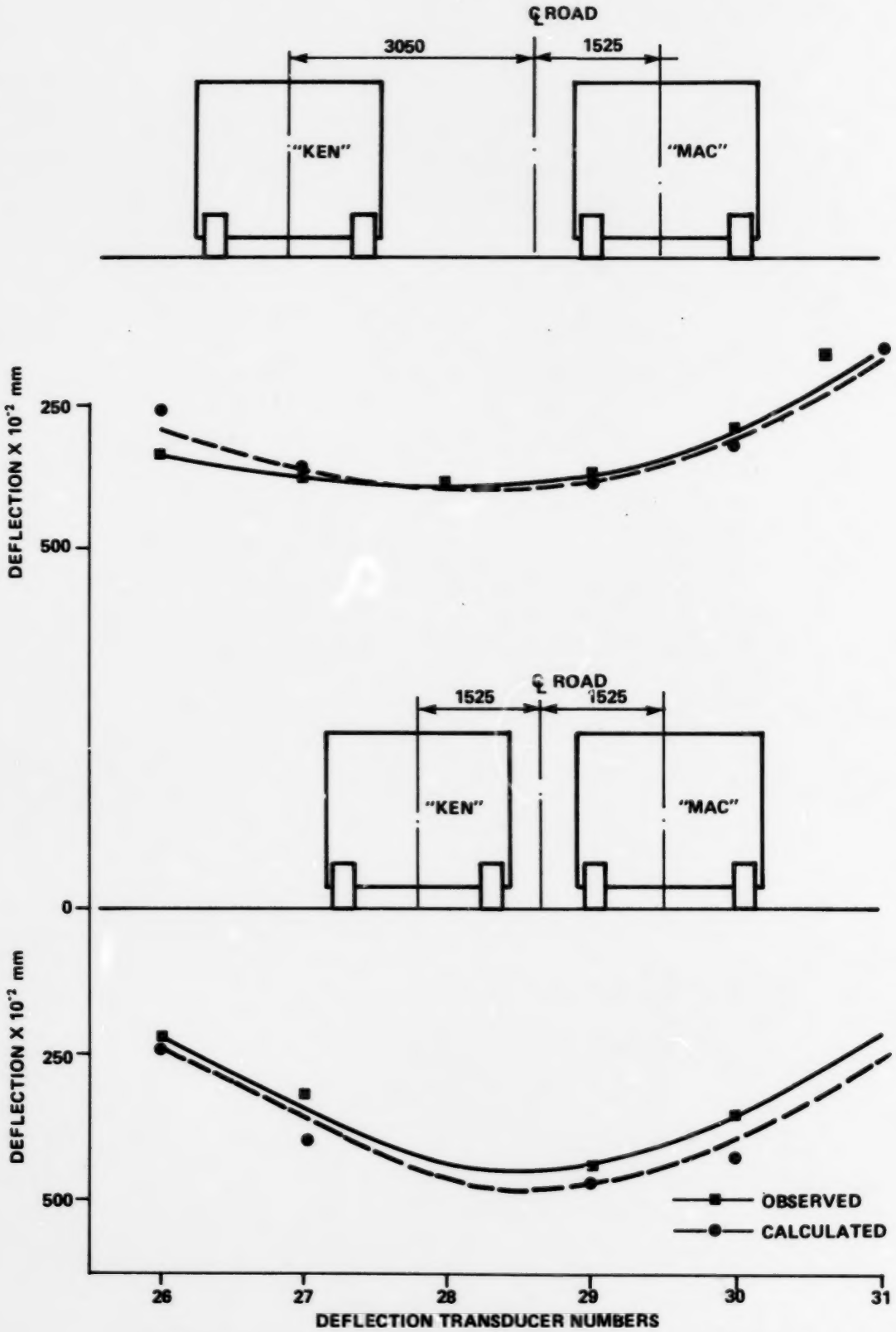


Figure 12/ Comparison of Maximum Observed Deflections and Calculated Deflections at Interior Mid-span Due to the Trucks Weighing 665 kN (48 Blocks Each)

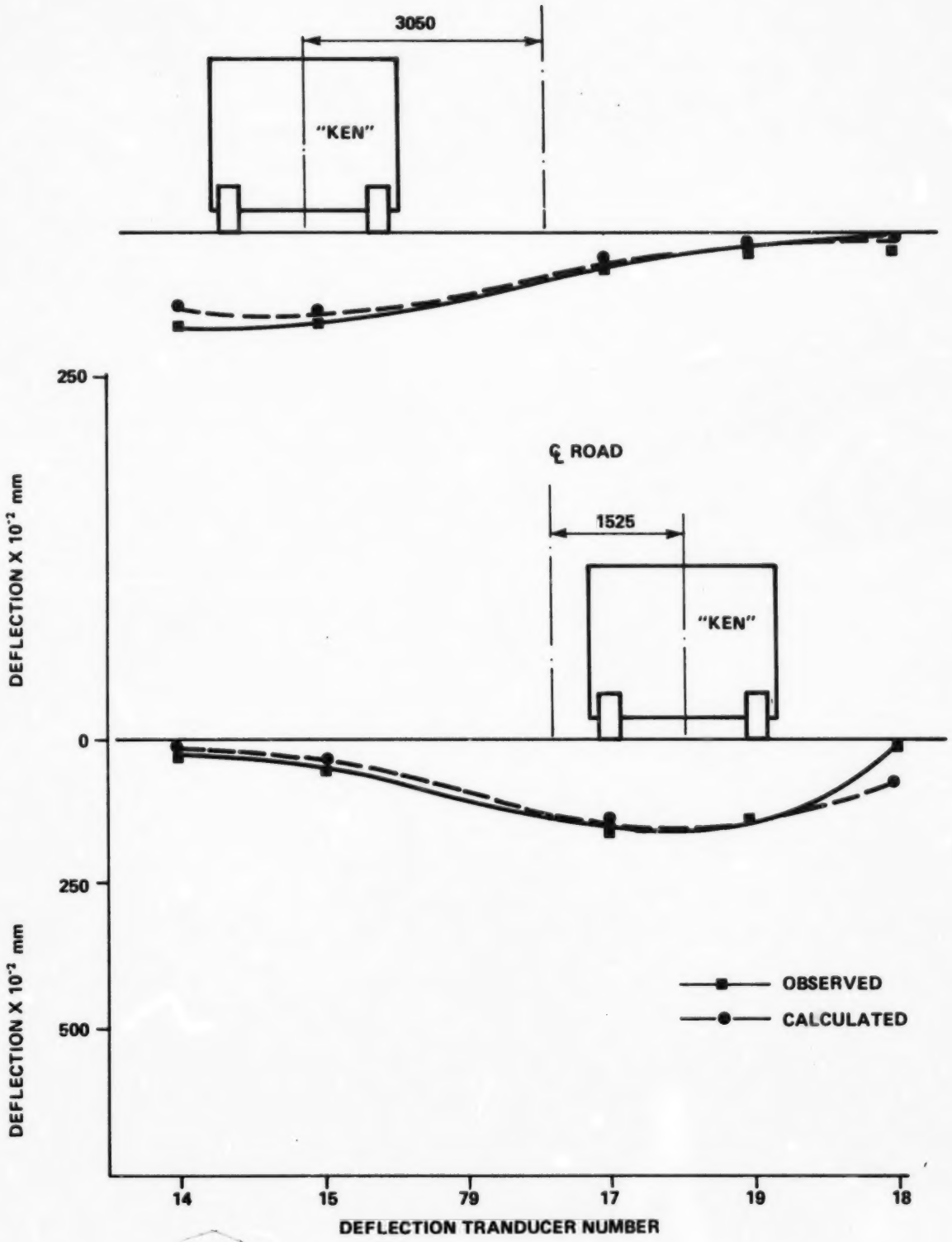


Figure 13/ Comparison of Maximum Observed Deflections and Calculated Deflections at Exterior Mid-span Due to the Truck Load of 890 kN (72Blocks)

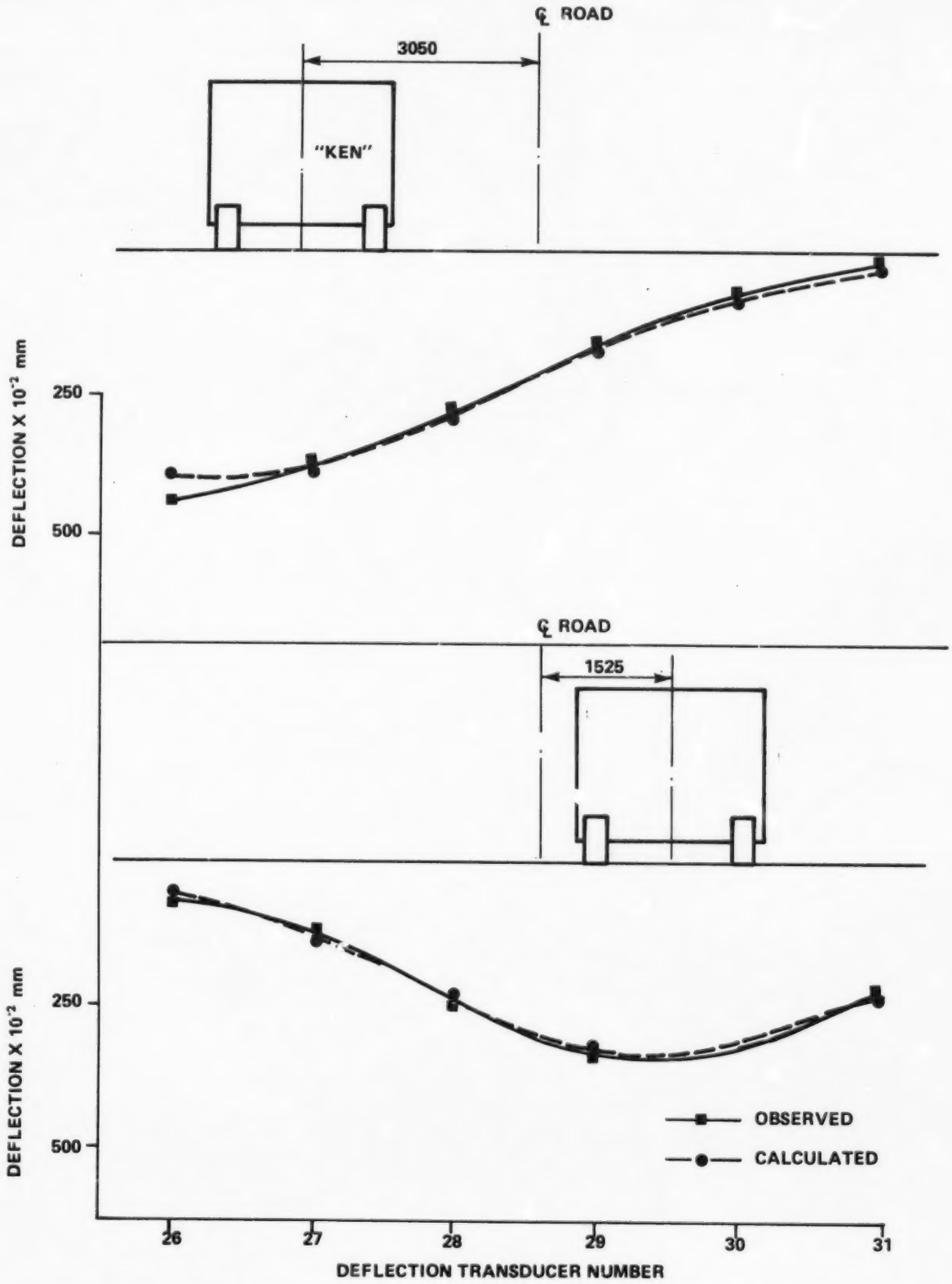


Figure 14/ Comparison of Maximum Observed Deflections and Calculated Deflections at Interior Midspan Due to Truck Load of 890 kN (72 Blocks)